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Dissipative Capacity Analysis of Steel Buildings using Viscous Bracing Device

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***Abstract*— Energy dissipation Systems in civil engineering structures are sought when it comes to removing unwanted energy such as instability, earthquake and wind. Among these systems, there is the combination of structural steel frames with passive energy dissipation provided by Fluid Viscous Dampers (FVD). This device is increasingly used to provide better seismic protection for existing as well as new buildings and bridges. A 3 D numerical investigation is done considering the seismic response of a twelve-story steel building moment frame with diagonal FVD that have linear force versus velocity behaviour. Nonlinear time history, which is being calculated by Fast nonlinear analysis (FNA), of Boumerdes earthquake (Algeria, May 2003) is considered for the analysis and carried out using the SAP2000 software and comparisons between unbraced, braced and damped structure are shown in a tabulated and graphical format. The results of the various systems are studied to compare the structural response with and without this device of the energy dissipation thus obtained were discussed. The conclusions showed the formidable potential of the FVD to improve the dissipative capacities of the structure without increasing its rigidity. It is contributing significantly to reduce the quantity of steel necessary for its general stability.**

***Index Terms*— Steel structure, bracing, energy dissipation, viscous fluid damper, seismic analysis.**

I. Introduction

Man has always lived with earthquakes. Some of them are so small that they are not felt, others against, are so strong that they can destroy an entire city and cause major damage in infrastructures (bridges, buildings, etc...) and kill thousands of people.

During a seismic event, the input energy from the ground acceleration is transformed into both kinetic and potential (strain) energy which must be either absorbed or dissipated through heat. However, for strong earthquakes a large portion of the input energy will be absorbed by hysteretic action (damage to structure). So for many engineers, the most conventional approach to protect the structures (buildings and bridges) against the effects of earthquakes is to increase the stiffness. This approach is not always effective, especially when it is an environment that promotes resonance and amplification of seismic forces.

To do this, the field of the earthquake engineering has made significant inroads catalyzed by the development of computational techniques on computer and the use of powerful testing facilities. This has favoured the emergence of several innovative technologies such as the introduction of special damping devices in the

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structures, which have the immediate effect of increasing the critical damping ratio right up 20 to 30% (against 5% value usually used for metal structures) and at the same time reducing the stresses and strains generated by earthquakes. This approach, is commonly known as the "energy dissipation", and has the capacity to absorb significant efforts without damaging the structure and ensuring the protection of human lives and property [1].
This approach of seismic energy dissipation is made clear by considering the following time-dependent conservation of energy relationship [2].

$$E(t) = E_k(t) + E_s(t) + E_h(t) + E_d(t) \quad (1)$$

Where:
E is the absolute energy input from the earthquake motion;
E_k is the absolute kinetic energy.
E_s is the elastic (recoverable) strain energy, E_h is the irrecoverable energy dissipated by the structural system through inelastic or other forms of action (viscous and hysteretic)
E_d is the energy dissipated by the supplemental damping system and t represents time.
The absolute input energy, E, represents the work done by the total base shear force at the foundation on the ground displacement and thus accounts for the effect of the inertia forces on the structure.
In the conventional design approach, the term E_d in equation (1) is equal to zero. In this case acceptable structural performance is accomplished by the occurrence of inelastic deformations, which has direct effect of increasing E_h. Finally the increased flexibility acts as filter which reflects a portion of seismic energy.
Introduction of supplemental damping devices in the structure involves increasing the term E_d in equation (1) and accounts for the major seismic energy that is absorbed during the earthquake.
In a supplemental dissipation energy system, mechanical devices are incorporated in the frame of the structure or within the base isolation system (Fig. 1).
Among these devices, there is the Fluid Viscous Dampers (FVD) which is included in the passive control systems of structural response. These systems have the ability to transmit developed forces according to the request of the structural response. Passive control devices dissipate energy in the structure, but can not increase it. Because of their great ability to return a building to its original position after an earthquake, they are increasingly used in the bracing structures in civil engineering in general and in the metallic high-rise structures in particular. The additional cost of the damper is typically offset by the savings in the steel weight and foundation concrete volume [3]. This device and its effect on the seismic structure response are the subject of this study.

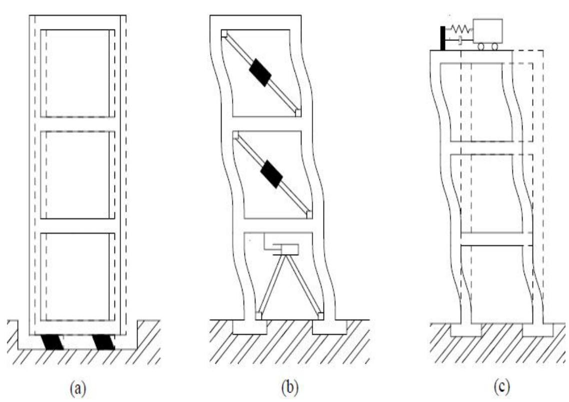


Figure1. Passive response control system:
a) Seismic isolation b) FVD c) Dynamic vibration absorber [4]

II. FLUID VISCOUS DAMPER

Fluid viscous dampers were designed for use in structural engineering in the early of 1990's. FVD typically consist of a piston head with orifices contained in a cylinder filled with a highly viscous fluid, usually a compound of silicone or a similar type of oil. Energy is dissipated in the damper by fluid orificing when the

piston head moves through the fluid [5]. The fluid in the cylinder is nearly incompressible, and when the damper is subjected to a compressive force, the fluid volume inside the cylinder is decreased as a result of the piston rod area movement. A decrease in volume results in a restoring force. This undesirable force is prevented by using an accumulatorwhich works by collecting the volume of fluid that is displaced by the piston rod and storing it in the make-up area. As the rod retreats, a vacuum that has been created will draw the fluid out. A damper with an accumulator is illustrated in figure 2 [6].

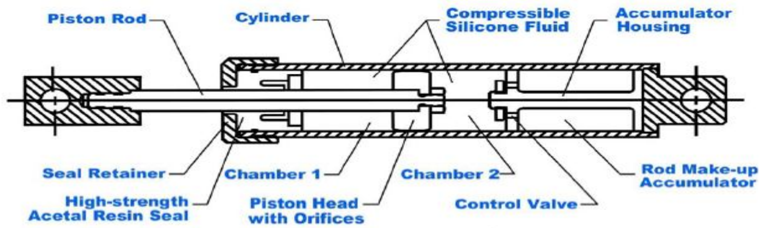


Figure2. Fluid Viscous Dampers (FVD)

A. Characteristics of Fluid Viscous Dampers

FVD are characterized by a resistance force F. It depends on the velocity of movement, the fluid viscosity and the orifices size of the piston. The value of P given by the relationship (Fig. 4) [7]:

$$P = C_d.\left(u_d^\bullet\right)^\alpha.\sin\left(u_d^\bullet\right) \qquad (2)$$

With: $$u_d(t) = u_0.\sin(\omega.t) \qquad (3)$$

$u^\bullet$ is the velocity between two ends of the damper and C_d is the damping constant;
u_0 is the amplitude of the displacement, ω is the loading frequency, and t is time;
α is an exponent which depends on the viscosity properties of the fluid and the piston.
The value of the constant α may be less than or equal to 1. Figure 3 shows the force velocity and the force displacement relationships for three different types of FVD. It characterizes the behaviour of the viscous damper. With $\alpha = 1$ the device is called linear viscous damper and for $\alpha < 1$ non-linear FVD which is effective in minimizing high velocity shocks. Damper with $\alpha > 1$ have not been seen often in practical application. The non-linear damper can give a larger damping force than the two other types (Fig. 3) [8].

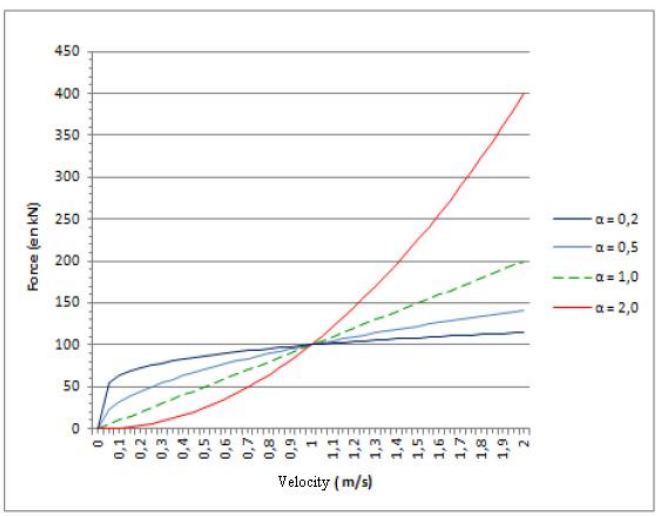


Figure 3. Force Velocity relationship of FVD

FVD allow very significant energy dissipation where the stress - strains diagram show a hysteretic loop approaching an ellipse for a pure viscous linear behaviour. The absence of storage stiffness make the natural's frequency of the structure incorporated with the damper remains the same. This advantage will

simplify the design procedure with supplemental viscous devices. However if the damper develops restoring force the loop will be changer from figure 4.a to 4.c while Figure 4.b shows the structure's behaviour without dampers. It turns from viscous behaviour to viscoelastic behaviour. The maximum energy amount that this type of damper can dissipate in a very short time is only limited by the thermal capacity of lead and steel tube.

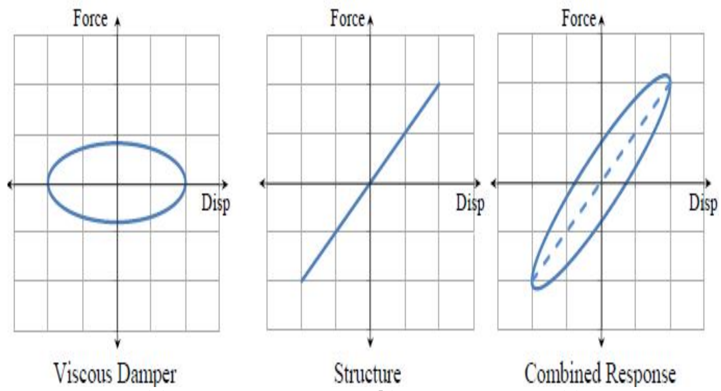


Figure 4. Hysteretic curve of FVD [9]

B. Analytical Model of the Fluid viscous damper

Fluid viscous dampers exhibit a viscoelastic behaviour, which can be best predicted with the Kelvin and Maxwell models for linear and non linear models respectively (Fig. 5) [10].

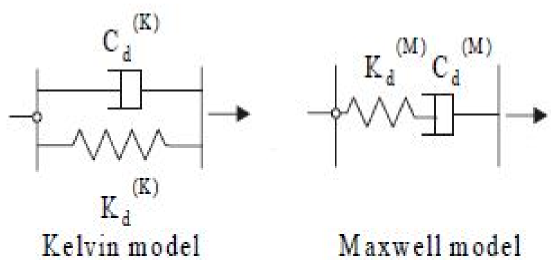


Figure5. Maxwell model

The model can also be described by the following equation:

$$P(t) + \lambda \frac{dP(t)}{dt} = C_d . \frac{du_d}{dt} \qquad (4)$$

Where: $u_d(t) = u_0 . \sin(\omega t)$

P is the damper output force, λ is the relaxation time, $C_d$is the damping constant at zero frequency, and u is the displacement of the piston head with respect to the damper housing. The relaxation time for the damper is defined as

$$\lambda = \frac{C_d}{K_1} \qquad (5)$$

Where $K_1$is the storage stiffness of the damper at infinite frequency.

For identification of the damper behaviour, the classical Maxwell model of equation (4) was generalized to the following form in which the derivatives are of fractional order [11].

$$P(t) + \lambda.D^r[P(t)] = C_d.D^q[u(t)] \qquad (6)$$

Where:

$D^r[P(t)]$ and $D^q[u(t)]$ are fractional derivatives of orders r and q, which are based on material properties. For complex viscoelastic behaviour, the fractional derivative model typically offers an approved ability to describe the damper behaviour over a wide frequency range. Other more advanced models of viscoelasticity have been examined for modelling the behaviour of fluid damper. For example Makris et al. [12] examined an even more advanced model of viscoelasticity to study the behaviour of fluid dampers. In this model the order of the time derivatives and the coefficients are complex-valued. The resulting models may be regarded as simplified forms of linear models of viscoelasticity.

B.1 Linear fluid viscous damper

The current study focused on linear fluid viscous damping. The model described by Equation (6) can be simplified to obtain a more useful model of linear viscous damping. When r=q=1 the model becomes the Maxwell model described by Equation (4). The device parameters, λ and C_d, were obtained from experimental tests performed in studies by Constantinou and Symans [13]. If the frequency of vibration is below the cut-off frequency, the second term in Equation (6) drops out and the model of the damper can be simplified as:

$$P(t) = C_d.\frac{du_d}{dt} \qquad (7)$$

Where:
C_d is independent of the frequency, but dependent on ambient temperature.
The energy dissipated by damper is [14]:

$$W_D = \oint F_D.du \qquad (8)$$

$$\Rightarrow W_D = \pi.C_d.u_0^2.\omega \qquad (9)$$

Recognizing that the damping ratio contributed by the damper can be expressed as $\xi_d = C_d / C_{cr}$ is obtained and natural excitation frequency is $\omega_0 = 2.\pi / T = \sqrt{K / M}$.

$$\Rightarrow W_D = 2.\pi.\xi_d.W_s.\frac{\omega}{\omega_0} \qquad (10)$$

Where: C_{cr}, K, M, ω_0 and W_s are respectively the critical damping coefficient, stiffness, mass, natural frequency and elastic strain energy of the system. The damping ratio attribute to the damper can then expressed as

$$\xi_d = \frac{W_D.\omega_0}{2.\pi.W_s.\omega} \quad (11)$$

Under earthquake excitation, ω is essentially equal to ω_0 and the equation (11) is reduced to

$$\xi_d = \frac{W_D}{2.\pi.W_s} \quad (12)$$

C. Modeling of system with Fluid viscous damper

The figure 6 shows a structure with a multiple degrees of freedom connected to FVD. The motion equation of the structure subjected to a ground vibration becomes [4]:

$$[M]U^{\bullet\bullet}+[(C)]U^{\bullet}+[(k\ )]U+F_d(t)=-[M].x_g^{\bullet\bullet} \quad (13)$$

Where:

M: Structure mass

K = Structure equivalent stiffness

C: Damping coefficient of the structure

$F_d(t)$: FVD force vector

$U, U^{\bullet}, U^{\bullet\bullet}$: Displacement, velocity and acceleration vectors of the structure.

$x_g^{\bullet\bullet}$: Ground acceleration (Earthquake).

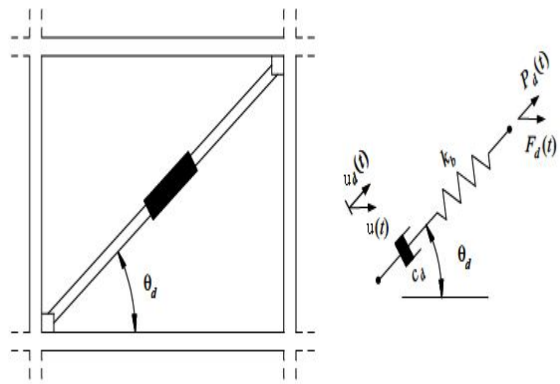


Figure 6. Multiple Degrees of Freedom (SDOF) structure with FVD

Considering a MDOF system as shown in figure 6 the total effective damping ratio of the system ξ_{eff} , is defined as

$$\xi_{eff} = \xi_0 + \xi_d \quad (14)$$

Where:

ξ_0 is the inherent damping ratio of the MDOF without dampers, and ξ_d is the damping ratio of the FVD. Extended from the concept of SDOF system, the equation (15) is used by FEMA273 (Federal Emergency Management Agency) [15] to represents ξ_d .

$$\xi_d = \frac{\sum W_j}{2.\pi.W_K} \quad (15)$$

$\sum W_j$ is the sum of the energy dissipated by the j th damper of the system in one cycle; and W_K is the elastic strain energy of the frame. W_K is equal to $\sum F_i.U_i$ where F_i is the story shear and U_i is the story drift of the i th floor. Now the energy dissipated by the FVD can be expressed by

$$\sum_j W_j = \sum_j \pi.C_j.u_j^2.\omega_0 = \frac{2.\pi^2}{T}.\sum_j C_j.u_j^2 \quad (16)$$

Where: u_j is the relative axial displacement between two ends of the damper.

Usually, only the first mode of The MDOF system is considered in the simplified procedure of practical applications. Using the modal strain energy method, theenergy dissipated by dampers and elastic strain energy provided by the structure without FVD can be rewritten

$$\sum_j W_j = \frac{2.\pi^2}{T}.\sum_j C_j.\phi_{rj}^2.\cos^2\theta_j \quad (17)$$

And:

$$W_K = \Phi_1^T.[K]\Phi_1 = \Phi_1^T.\omega^2.[M]\Phi_1$$

$$W_K = \sum_i \omega^2.M_i\phi_i^2 = \frac{4.\pi^2}{T}.\sum_i M_i.\phi_i^2 \quad (18)$$

Where:

$[K]$, $[M]$ and Φ_1 are respectively, stiffness matrix, mass matrix and first mode shape of system; ϕ_{rj} is the relative horizontal displacement of damper j corresponding to first mode shape. ϕ_i is the first mode shape at floor and θ_j is the inclined angle of the damper j. Substituting equations (16), (17) and (18) into (14), the ξ_{eff} of a structure with linear FVD is given by:

$$\xi_{eff} = \xi_0 + \frac{T.\sum_j C_j.\phi_{rj}^2.\cos^2\theta_j}{4.\pi^2.\sum_i M_i.\phi_i^2} \quad (19)$$

There is no substantial procedure suggested by the design codes for distributing C values over the whole buildings. When designing the dampers, it may be convenient to distribute C values equally in each floor.

III. Case Study

A. *Structure characteristics*

A twelve-story steel building modelled as 3D moment resisting frame is analyzed with and without viscous dampers using SAP2000 [17]. The profiles of the various frame elements are shown in figure 7. The properties of the building and related information are given in table I.

TABLE I. GEOMETRIC PROPERTIES OF BUILDING

Geometric properties of building	
Total length	23.70 m
Total Width	22.92 m
Total Height	45.82 m
Height of 3rd floor	3.40 m
Height of other floors	4.42 m
Modulus of Elasticity	200 GPa
Steel weight per unit volume	7698 KN/m^3
Poisson ratio	0..3

The damper stiffness inserted into the SAP2000 model is equal to one diagonal of L120x13 profile.

The lateral dynamic load applied to the structure was simulated by nonlinear time history (FNA) of the Boumerdes earthquake (Algeria May 2003). This gives in the form of text file having 7000 points of acceleration data equally spaced at 0.05 second. The use of Nonlinear time history (NLTH) analysis is mandated for most passively damped structures because the earthquake vibration of most civil engineering structure will induce deformation in one or more structural element beyond their yield limit. Therefore, the structure will respond with a nonlinear relationship between force and deformation. The results were summarized in the following paragraph.

B. Results and interpretation

To maximize the performance of a dampers, upstream optimization study on the location of diagonal steel bracing (cross brace with L120x13 angle profile) positions was conducted on twelve alternatives (Fig. 8).

The results show that the alternative No. 10 was the best in vibration's period, mass participation and satisfy all conditions of RPA99/2003 [18]. It was compared with non-braced and damped models (Table2).

As expected, the fundamental period of vibration for the braced structure decreases due to the increased stiffness. In the third case, the period decreases due to the added stiffness resulting from the use of dampers.

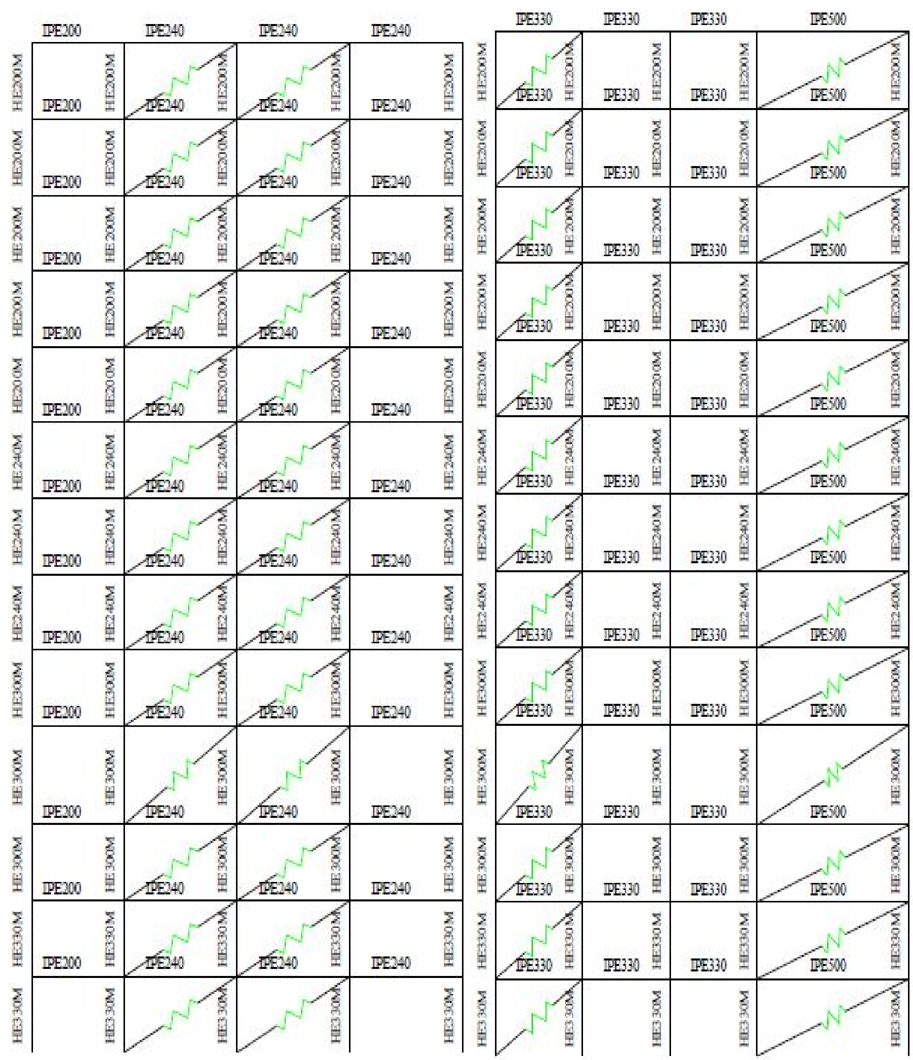


Figure 7. Modelling of twelve-story building connected to FVD

TABLE II. RESULTS COMPARISON OF THE THREE MODELS

Unbraced structure		Braced structure (cross)		FVD Damped structure (FVD)	
Period (s)	M .P. (%)	Period (s)	M .P. (%)	Period (s)	M .P. (%)
$T_1 = 7.47$	76.36	$T_1 = 2.02$	73.13	$T_1 = 2.32$	77.87
$T_2 = 4.84$	75.50	$T_2 = 1.87$	76.21	$T_2 = 2.31$	75.00
$T_3 = 3.95$	76.13	$T_3 = 1.33$	77.77	$T_3 = 1.67$	74.65

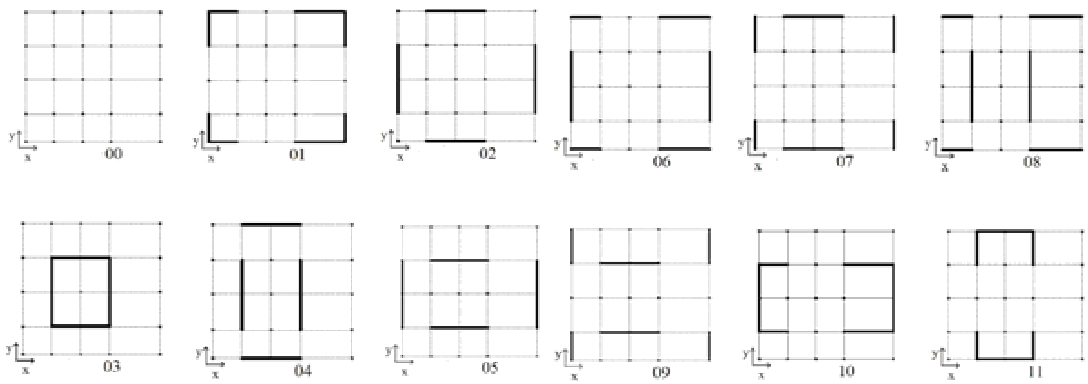


Figure 8. The different location of diagonals alternatives

It should be noted that the number of diagonals used in the third case is reduced by half compared to the second case however the values of the periods remain close. The time history analysis of top displacement and acceleration response of the structure in the three models (Fig. 9& 10) shows a significant response decrease of the structure equipped with FVD, when compared to the unbraced ($\xi_{eff} = 5\%$) design. When the top displacement value of the unbraced structure reach maximum, the corresponding one of the damped structure ($\xi_{eff} = 35\,\%$) decreases by 32%.

It also can be observed that the acceleration response of the two cases, damped and self-supporting is almost the same. It means that the increase of structure stiffness with the addition of the supplement dampers hasn't increase its acceleration; unlike the comparison with cross braces case where the model of the FVD response decreases at the peak by 37%.This can lead to reduce the unpleasant effects of acceleration for occupants of these structures but also for non-structural parts, pipes, false ceilings, etc.

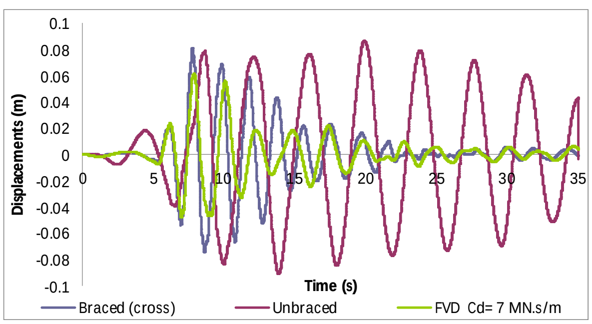


Figure 9. Time history displacement response

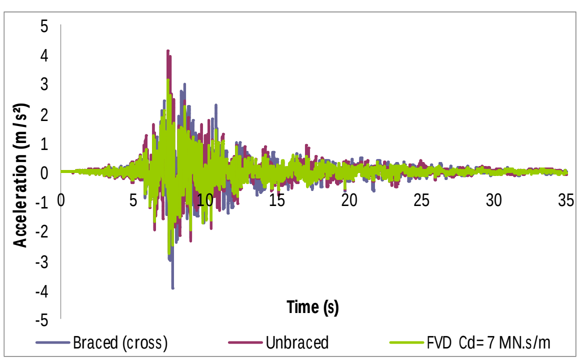
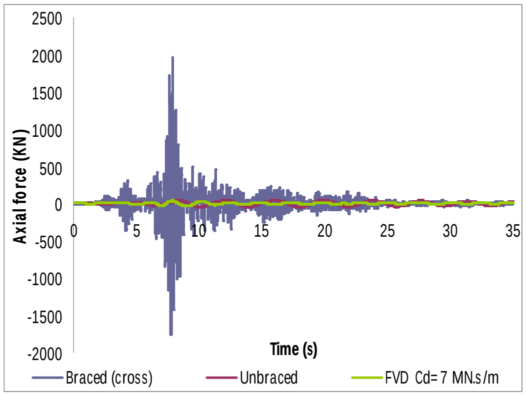
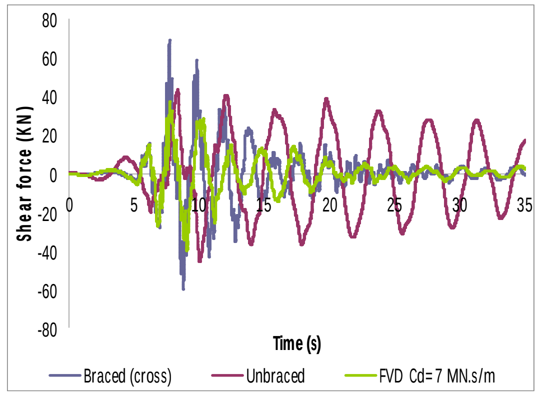


Figure10. Time history acceleration response(Cross braced, unbraced and structure with AFV)

The verification of structural members' stability is checked in combinations including earthquake (RPA99-2003 section 5.2), however a time history analysis of the top axial (N), shear (V) forces and moment (M) of the seismic loading has been carried out (Fig. 11) [19]. The results showed a decrease values for reinforced cross brace and FVD models with a net benefit to the dissipative device model. This decrease is due to the additional stiffness provided by the reinforcing elements but it is also due to the increase of damping rate for the FVD model. It is also important to note that in the braced structure, the cross diagonals transmit a very important axial force to its near columns, valued at 51 times the ones of the damped model. This last has the ability de decrease them by developing a resisting forces induced by the dampers.





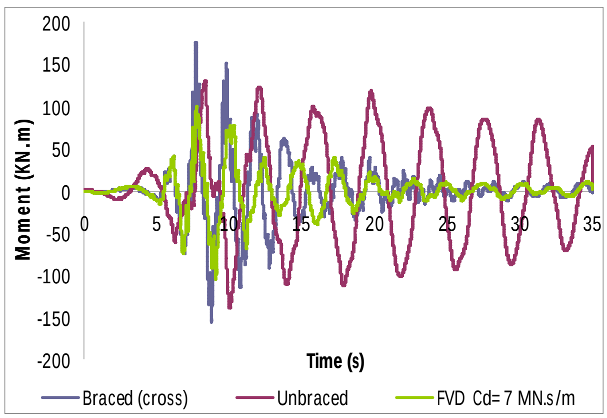


Figure 11. Time history variation of N, T, M in the most loaded column

Figure 12 gives a particularly interesting reflection on the ability of FVD to reduce the base shear force. Note that it becomes very important in the cross braced case. It is due to the decrease of the fundamental period (T=2.02 sec) which makes greater acceleration but this forces decrease rapidly over time due to the stiffness of the system. Unlike to the unbraced model where the base shear force is not very important (T=7.47 sec) but remains constant throughout the duration of the signal. In the third model, forces are also low (T = 2.32sec) and they disappear quickly and completely after 15 sec of vibration. This is due to the capacity of FVD to produce a passive control system by balancing quickly the load forces to the resistance and damping forces.

The figure 13 illustrates the variation of the axial force (N) according to the FVD damping constant C_d, for X and Y directions of earthquake. The curves have shown an exponential pace that can be compared to two straight lines. The first line shows a decreasing force versus to an increasing of damping constant until the intersection with the second line where the values become almost constant. We can conclude that for C_d = 25

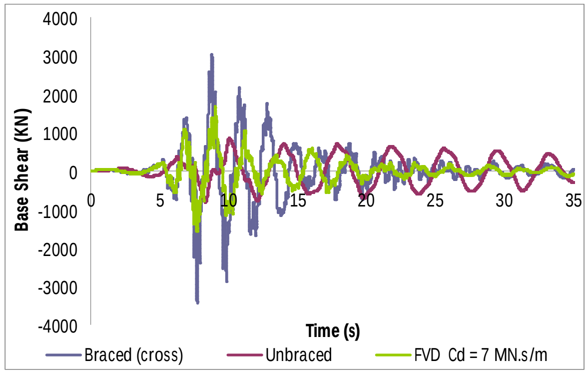


Figure 12. Time history variation of base shear force

MN.s/m the damped structure, can fully absorb the input energy of the seismic signal and supplement damping will not affect the system which will be already completely dissipated.This conclusion was confirmed by the results summarized in the table III which shows the variation of the FVD damping ratio ξ_d according to its damping coefficient C_d. It may be seen in the table that the increase of C_d generates an augmentation of the damping capacity of the structure to resist to the seismic, up to a value of ξ_d = 100%.

The results shown below are in accordance with those found by Lin and Chen [20] who conducted experiments on shaking table to verify the numerical model computed by sap 2000.

TABLE III. VARIATION OF ξ_d according to C_d

C_d (KN.MN.s/m)	W_s (Joule)	W_d(Joule)	ξ_d (%)
0	168.63	0	0
500	149,58	28.4	3
2000	119.74	75.63	10
4000	98	113.63	18.46
7000	78.31	150.82	30.5
10000	65.7	176.06	42.67
15000	52.14	204.33	62.5
20000	43.44	223.31	81.6
25000	37.37	237.14	100

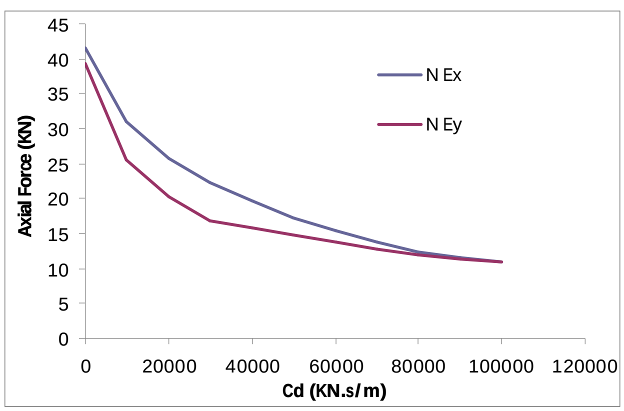


Figure 13. Variation of N versus Damping Constant

The curves of the figures 14 and 15 where the variation of the input and the damping energies of the system were compared for the values of C_d = 0 (Fig. 14) and C_d = 7 MN.sec / m (Fig. 15); show that an addition of

supplemental dampers results an increase of the absolute input energy. This is not surprising since at the end of theearthquake the absoluteinput energy mustequal to the dissipated energy in the system (Fig. 15).

Therefore, it is expected that the structure with dampers ($\xi_{eff} = 35\%$) would have a large absolute input energy at the end of the earthquake. Since the input energy at time t is the integral of the base shear over the ground displacement as it’s described in the following equation.

$$E(t) = \int_0^t m.\left[u^{\bullet\bullet}(t) + u_g^{\bullet\bullet}(t)\right]du_g(t) \quad (20)$$

So as mentioned below the increase of stiffness in the structure increases relatively the base shear in the damped model and consequently develops itsin put energy. In contrast, the undamped structure with its relatively low inherent damping ($\xi_{eff} = 5\%$); has low ability to dissipate load energy which results in a small absolute input energy at the end of earthquake. These results are comparable to those achieved by other works [6] . However the energy of the seismic signal is completely dissipated by the addition of the modal damping (Ws) and link damping (W_D) energies. It means that the reduction in ductility demand is facilitated through displacement reductions that come with increased damping.

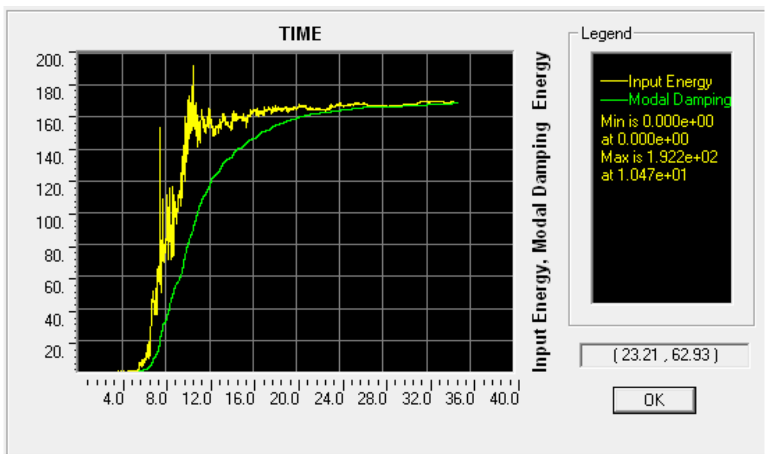


Figure 14 Variation of the input and damping energies of the system for C_d = 0 MN.sec / m

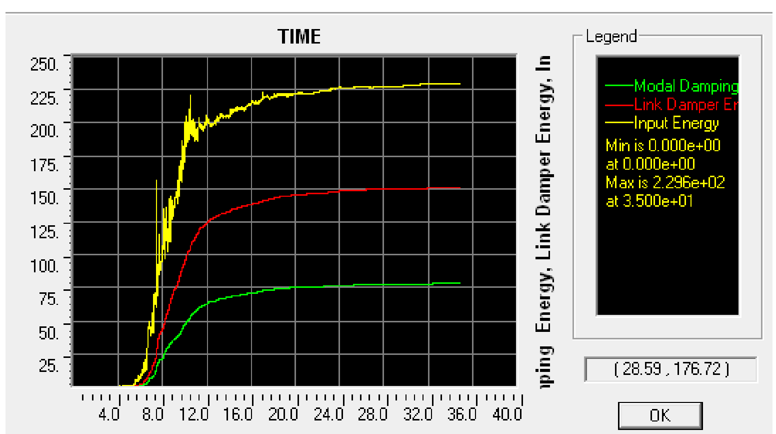
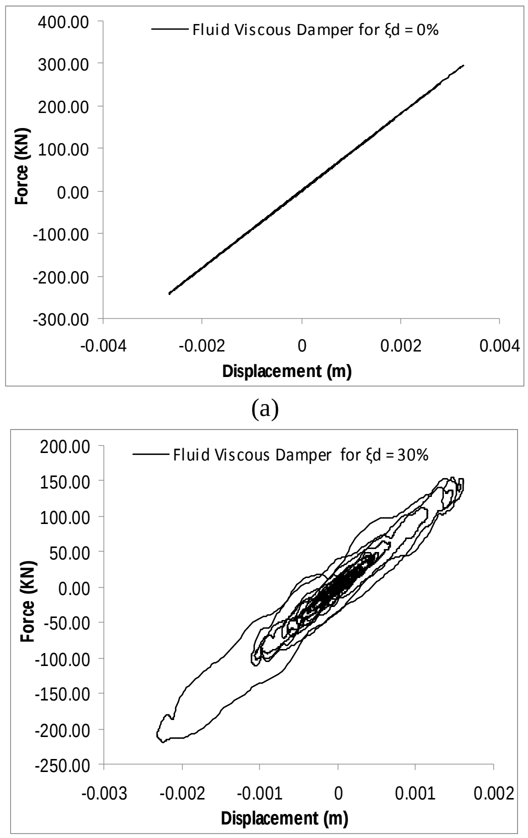


Figure15. Variation of the input and the damping energies of the system for C_d = 7 MN.sec / m

The analyses of the FVD damped structure are run with a range of different damping ratios. The plot of the figure 16 shows the hysteric loops of the dampers (placed at 7^{th} story of a building) response contributing with no supplemental damping ($\xi_d = 0\%$) (Fig. 16a) and 30% damping ($\xi_d = 30\%$) (Fig. 16b). It can be

seen that the structure in figure 16a has an elastic force displacement relationship; therefore its behaviour is comparable to a simply diagonal brace. One could observe that while the peak force occurs at different displacements it is within 20% higher compared to the partially damped structure (Fig. 16b).
It is also important to note that the peak force, which occurs at the peak velocity, does not occur at the zero displacement position, which is what would be expected from a standard harmonic response.
The curves shapes of the figure 16 are similar to the concept presented schematically in the figure 4. The results thus highlight the importance of considering the overall balance of damping added, even within realistic ranges of (overall) damping, and especially for cases of structures with augmented damping. Hence, it may be considered that these results justify the overall proof of concept analysis presented in this work.
As expected and as seen below the restoring force induced by the damper generate it viscoelastic behaviourwhich permit it greater capacity to dissipate the dynamic loading energies. This plot demonstrates the validity of the analytical model versus to those in the literature review [9].



(a)

(b)

Figure 16. Hysteric loops of damper resistance force versus displacement

The analyses of the coefficient of damping C_d distribution at the different stories of the building, has been considered in the curve of the figure 17. The results showed that the resisting forces generate by the dampers decrease according to the story height. It means that the damping coefficient used at the upper stories is greater than required for the dissipation of the motion. It induce that the upper dampers are used under there dissipation capacity. In contrast of the lower dampers which are very efficient. Those results are in accordance with Yang and al. [16] which demonstrates that the distribution of its damping ratio must decrease according to the height's building for more efficient use of this device and for economy.
Finally the analysis of inter-story drift curve according on the height of building was carried out for the three models. Results are shown in figure 18. The variationcurve of the damped structure with FVD takes almost looks like a vertical line whose values are almost constant. The result which is comparable with that achieved by Yazdan and al [21], shows that the structure has ones block behaviour.

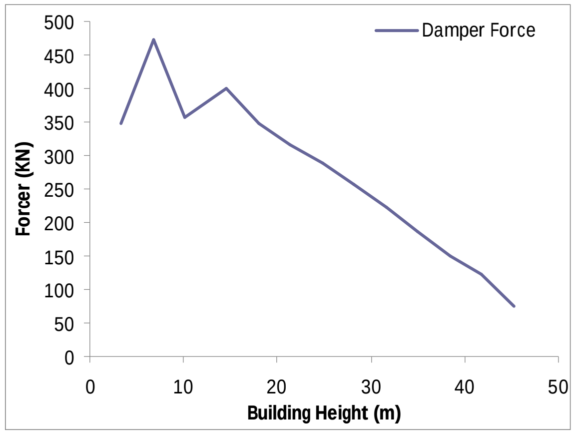


Figure 17. Damping force variation according to building height

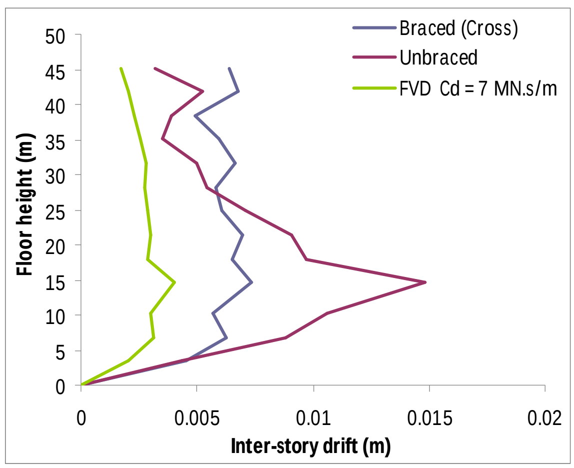


Figure 18. Inter-story drift variation versus building height

IV. Conclusions

This study permitted to analyse the difference in steel structure behaviour, with and without viscous damper fluid for a seismic load. Numerical calculation with SAP2000 software was used for the analysis of a 12-story building. The results show that the use of the passive control device FVD in buildings generates a very significant reduction of the structural response compared to the unbraced ones. However, in the case of a 12-storey building, the main conclusions are summarized below:

- The fundamental period decreases by 220% compared to the unbraced structure.
- The maximum displacements decrease of FVD model until 32% compared to the cross-braced structure
- Reduction of the maximum acceleration is 50%, which reduces values of base shear forces and its time loading
- Reducing efforts by more than 40% in bending moment and shear force in the most loaded member.
- With the damping energy dissipation, the diagonals do not transmit any undesirable axial forces.
- Beyond Cd = 25 MN.s/m, FVD cannot dissipate supplement seismic energy in the structure.
- The addition of supplemental dampers results an increase of the absolute input energy which is completely dissipated by the increase of damping energy W_D.
- The restoring force induced by the damper generate it viscoelastic behaviour which permit it greater capacity to dissipate the dynamic loading energies.

- The damping coefficient of the dampers used in structure and/or their number must decreases according to the height's building for more efficient use of this device and for economy
- The inter-storey drift become, almost zero, which generates block behaviour of the structure and reducing the effects of shear forces.

The benefits of energy dissipaters were clearly demonstrated by the comparison data and improving performance of the structure during an earthquake has been proven. The passive control system absorbs vibrations automatically and systematically. These devices are generally inexpensive and effective reinforcement of buildings subjected to dynamic excitations.

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